CHAPTER 9

FOUNDATIONS

9-1. Introduction.

Chapter 7 in FEMA 302 provides conventional foundation design provisions that are adequate for most military buildings. This chapter provides guidance in the implementation of those provisions, and also provides guidance in the use of load deformation characteristics, for soil/structure interaction, in the form of simplified soil springs. The determination of appropriate soil springs and the structural systems for which they provide a better representation of seismic response are discussed in Paragraph 9-2b.

9-2. Site Characterization.

Site characterization consists of the compilation of information on the site subsurface soil conditions, and the configuration and loading of the proposed foundations. The evaluation of the ground-shaking hazard and site geologic hazards is discussed in Chapter 3.

a. Site Foundation Conditions. Subsurface soil conditions must be defined in sufficient detail to assess the ultimate capacity of the foundation, and to determine if the site is susceptible to seismic-geologic hazards.

(1) Structural foundation type. Information regarding the structural foundation type, dimensions, and material are required irrespective of the subsurface soil conditions. This information includes:

- Foundation type: spread footings, mat foundation, piles, drilled shafts.

- Foundation dimensions: plan dimensions and locations. For piles, tip elevations, vertical variations (tapered sections of piles or belled caissons).

- Material composition/construction. For piles, type (concrete/steel/wood), and installation method (cast-in-place, open/closed-end driving).

(2) Subsurface soil conditions. The capacity of the foundation soil in bearing or the capacity of the soil interface between pile, pier, or caisson and the soil will be determined by a geotechnical investigation and shall be sufficient to support the structure with all prescribed loads, without seismic forces, taking due account of the settlement that the structure can withstand. For the load combination including earthquake, the soil capacities must be sufficient to resist loads at acceptable strains considering both the short duration of loading and the dynamic properties of the soil. If load-deformation characterization for the foundations are to be considered (i.e., Paragraph 9-2b), the soil unit weight, g soil shear strength, c, soil friction angle, f, soil compressibility characteristics, soil shear modulus, G, and Poisson's ratio, n, need to be determined for each soil type. Additionally, the shear wave velocity, $<_s$, the standard penetration resistance, N, or the undrained shear strength, S_u, need to be determined to define the site classification

in accordance with Table 3-1 in order to assign the appropriate site coefficients, F_a and F_v.

b. Load Deformation Characteristics for Foundations.

(1) General. Load-deformation characteristics are required where the effects of foundations are to be taken into account in linear elastic analyses or in nonlinear static (pushover) or nonlinear dynamic (time history) analyses. Foundation load-deformation parameters characterized by both stiffness and capacity can have a significant effect on both structural response and load distribution among structural elements. Load-deformation parameters, represented by appropriate soil springs, can provide significant reduction and/or redistribution of seismic force levels in some buildings. Vertical soil springs may effectively lengthen the fundamental period of slender, stiff buildings such as aircraft control towers, and could have a beneficial effect for buildings at relatively stiff soil sites. Rotational soil springs at interior column footings will tend to relieve the fixed end moment at the column base, and cause redistribution of seismic forces in the story. Foundation systems for buildings can in some cases be complex, but for the purpose of simplicity, three foundation types are considered in these guidelines:

- Shallow bearing foundations;
- Pile foundations; and
- Drilled shafts.

While it is recognized that the load-deformation behavior of foundations is nonlinear, because of the difficulties in determining soil properties and static foundation loads for existing buildings, together with the likely variability of soils supporting foundations, an equivalent elasto-plastic representation of load-deformation behavior is recommended. In addition, to allow for such variability or uncertainty, an upper- and lower-bound approach to defining stiffness and capacity is recommended (as shown in Figure 9-1a) to permit evaluation of structural response sensitivity. The selection of uncertainty represented by the upper and lower bounds should be determined jointly by the geotechnical and structural engineers.

(2) Shallow bearing foundations.

(a) Stiffness parameters. The shear modulus, G, for a soil is related to the modulus of elasticity, E, and Poisson's ratio, v, by the relationship:

$$G = \frac{E}{2(1+v)} \tag{9-1}$$

1. Most soils are intrinsically nonlinear and the shear modulus and the shear wave velocity decrease with increasing shear strain. Experimental values obtained by laboratory testing at low strains need to be modified to reflect expected effective values at strains corresponding to the design ground motion.

2. To reflect the upper- and lower-bound concept illustrated in Figure 9-1 the upper-bound stiffness of rectangular footings should be based on

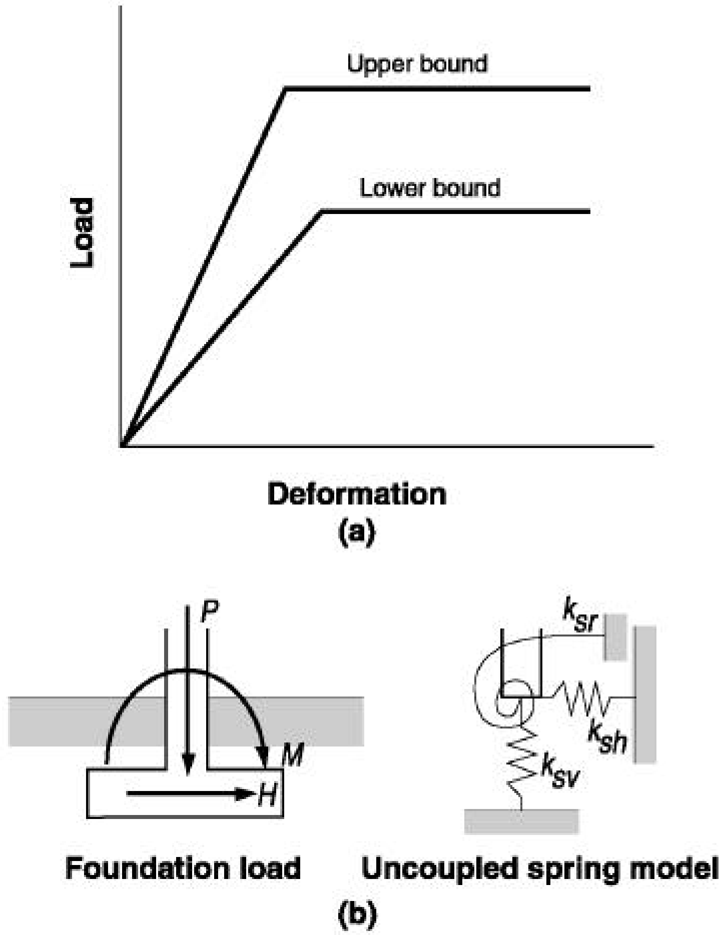


Figure 9-1 Idealized Elasto-Plastic Load-Deformation Behavior for Soils

twice the effective shear modulus, G, determined in the geotechnical investigation. The lower-bound stiffness should be based on one-half the effective shear modulus; thus, the range of stiffness should incorporate a factor of four from lower- to upper-bound. Most shallow bearing footings are stiff relative to the soil upon which they rest. For simplified analyses, an uncoupled spring model, as shown in Figure 9-1b, may be sufficient. The three equivalent spring constants may be determined using conventional theoretical solutions for rigid plates resting on a semi-infinite elastic medium. Although frequency-dependent solutions are available, results are reasonably insensitive to loading frequencies within the range of parameters of interest for buildings subjected to earthquakes. It is sufficient to use static stiffnesses as representative of repeated loading conditions. Figure 9-2 presents stiffness solutions for rectangular plates in terms of an equivalent circular radius. Stiffnesses are adjusted for shape and depth using factors similar to those in Figure 9-3. For the case of horizontal translation, the solution represents mobilization of base traction (friction) only. If the sides of the footing are in close contact with adjacent in situ foundation soil or well-compacted fill, significant additional stiffness may be assumed from passive pressure. A solution for passive pressure stiffness is presented in Figure 9-4. For more complex analyses, a finite element representation of linear or nonlinear foundation behavior may be accomplished using Winkler component models. Distributed vertical stiffness properties may be calculated by dividing the total vertical stiffness by the area. Similarly, the uniformly distributed rotational stiffness can be calculated by dividing the total rotational stiffness of the footing by the moment of inertia of the footing in the direction of loading. In general, however, the uniformly distributed vertical and rotational stiffnesses are not equal. The two may be effectively decoupled for a Winkler model using a procedure similar to that illustrated in Figure 9-5. The ends of the rectangular footing are represented by End Zones of relatively high stiffness, with overall length of approximately one-sixth of the footing width. The stiffness per unit length in these End Zones is based on the vertical stiffness of a B x $B/6$ isolated footing. The stiffness per unit length in the Middle Zone is equivalent to that of an infinitely long strip of footing. In some instances, the stiffness of the structural components of the footing may be relatively flexible compared to the soil material. A slender grade beam resting on stiff soil is an example. Classical solutions for beams on elastic supports can provide guidance regarding when such effects are important. For example, a grade beam supporting point loads spaced at a distance of L might be considered flexible if:

$$\frac{EI}{L^4} < 10 k_{sv} B \qquad (9\text{-}8)$$

where, for the grade beam,

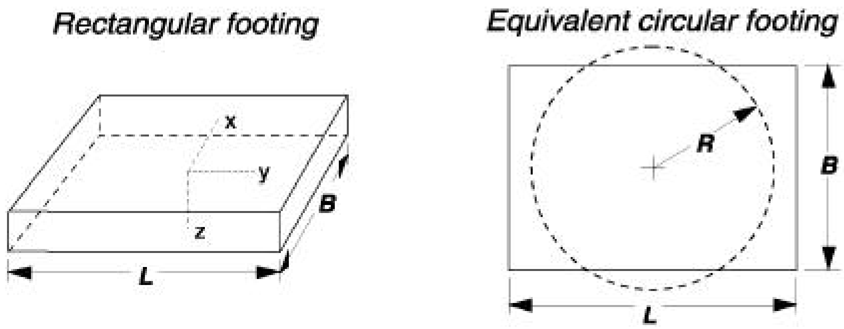
E = effective modulus of elasticity

I = moment of inertia

B = width.

For most flexible foundation systems, the unit subgrade spring coefficient, k_{sv}, may be taken as

Radii of circular footings equivalent to rectangular footings



	Degree of freedom			
	Translation	***Rocking***		***Torsion***
		About x-axis	*About y-axis*	*About z-axis*
Equivalent radius, R	$\left(\frac{BL}{\pi}\right)^{1/2}$	$\left(\frac{BL^3}{3\pi}\right)^{1/4}$	$\left(\frac{B^3L}{3\pi}\right)^{1/4}$	$\left[\frac{BL(B^2+L^2)}{6\pi}\right]^{1/4}$

Spring constants for embedded rectangular footings

Spring constants for shallow rectangular footings are obtained by modifying the solution for a circular footing, bonded to the surface of an elastic half-space, i.e., $k = \alpha\beta k_o$

where

- k_o = Stiffness coefficient for the equivalent circular footing
- α = Foundation shape correction factor (Figure 4-3a)
- β = Embedment factor (Figure 4-3b)

To use the equation, the radius of an equivalent circular footing is first calculated according to the degree of freedom being considered. The figure above summarizes the appropriate radii. k_o is calculated using the table below:

Displacement degree of freedom	k_o
Vertical translation	$\frac{4GR}{1-\nu}$
Horizontal translation	$\frac{8GR}{2-\nu}$
Torsional rotation	$\frac{16GR^3}{3}$
Rocking rotation	$\frac{8GR^3}{3(1-\nu)}$

Note:
G and ν are the shear modulus and Poisson's ratio for the elastic half-space. G is related to Young's modulus, E, as follows:
$E = 2(1+\nu)G$
R = Equivalent radius

Figure 9-2 Elastic Solutions for Rigid Footing Spring Constants (based on Gazetas, 1991 and Lam et al., 1991)

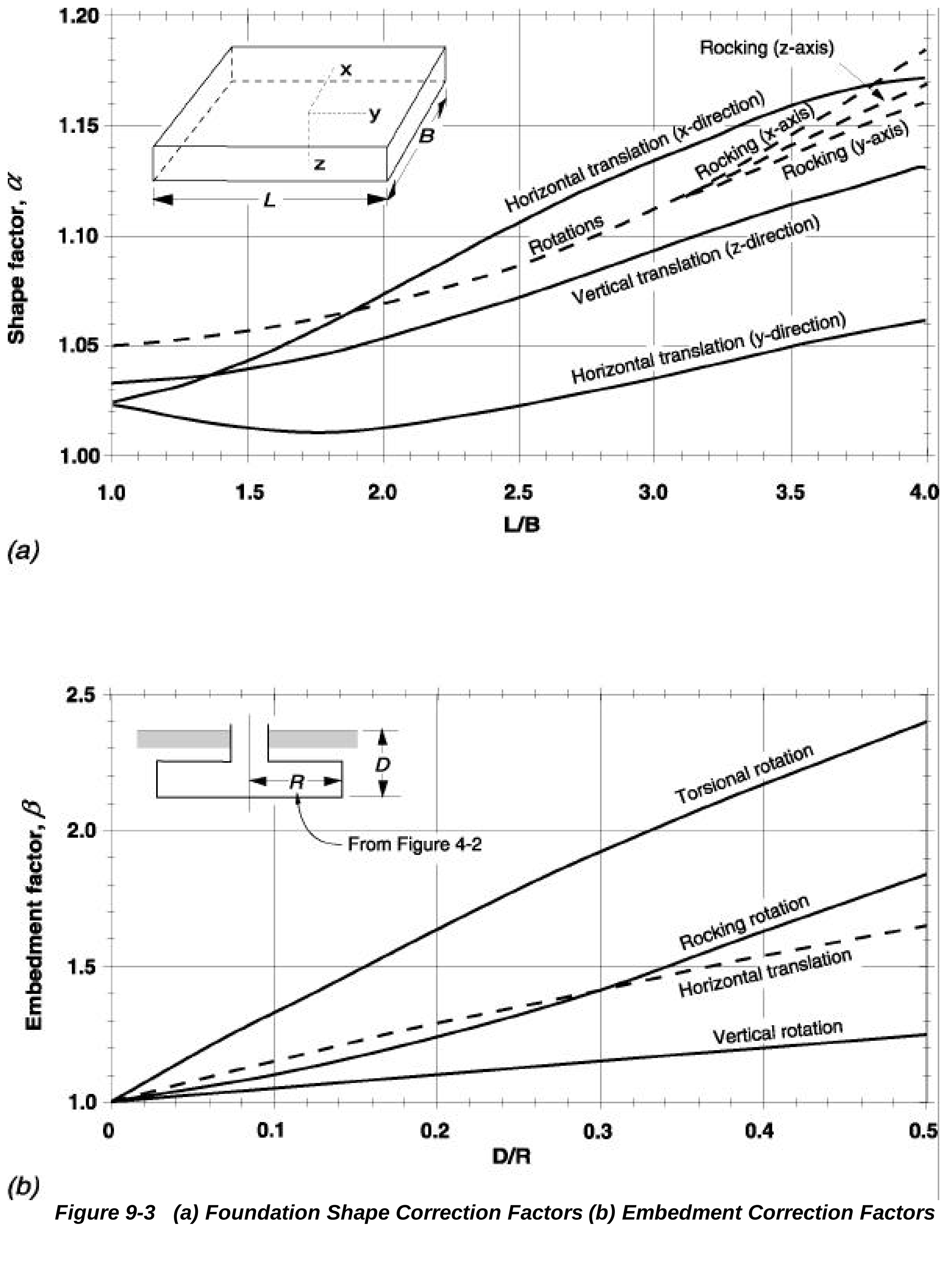


Figure 9-3 (a) Foundation Shape Correction Factors (b) Embedment Correction Factors

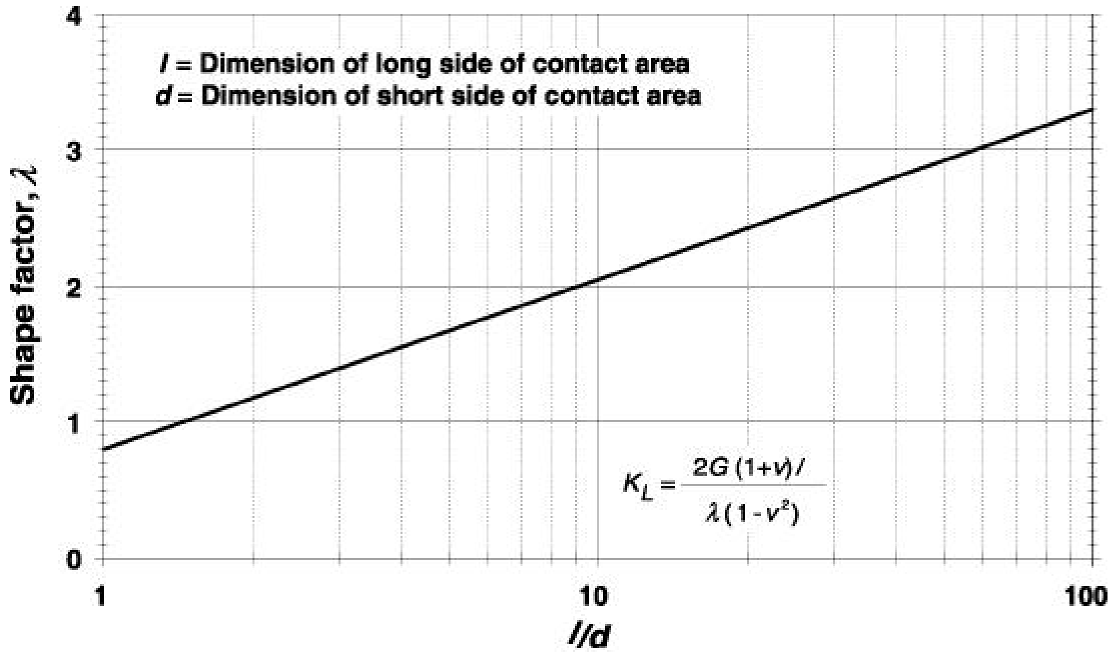


Figure 9-4 Lateral Foundation-Soil Stiffness for Passive Pressure (after Wilson, 1988)

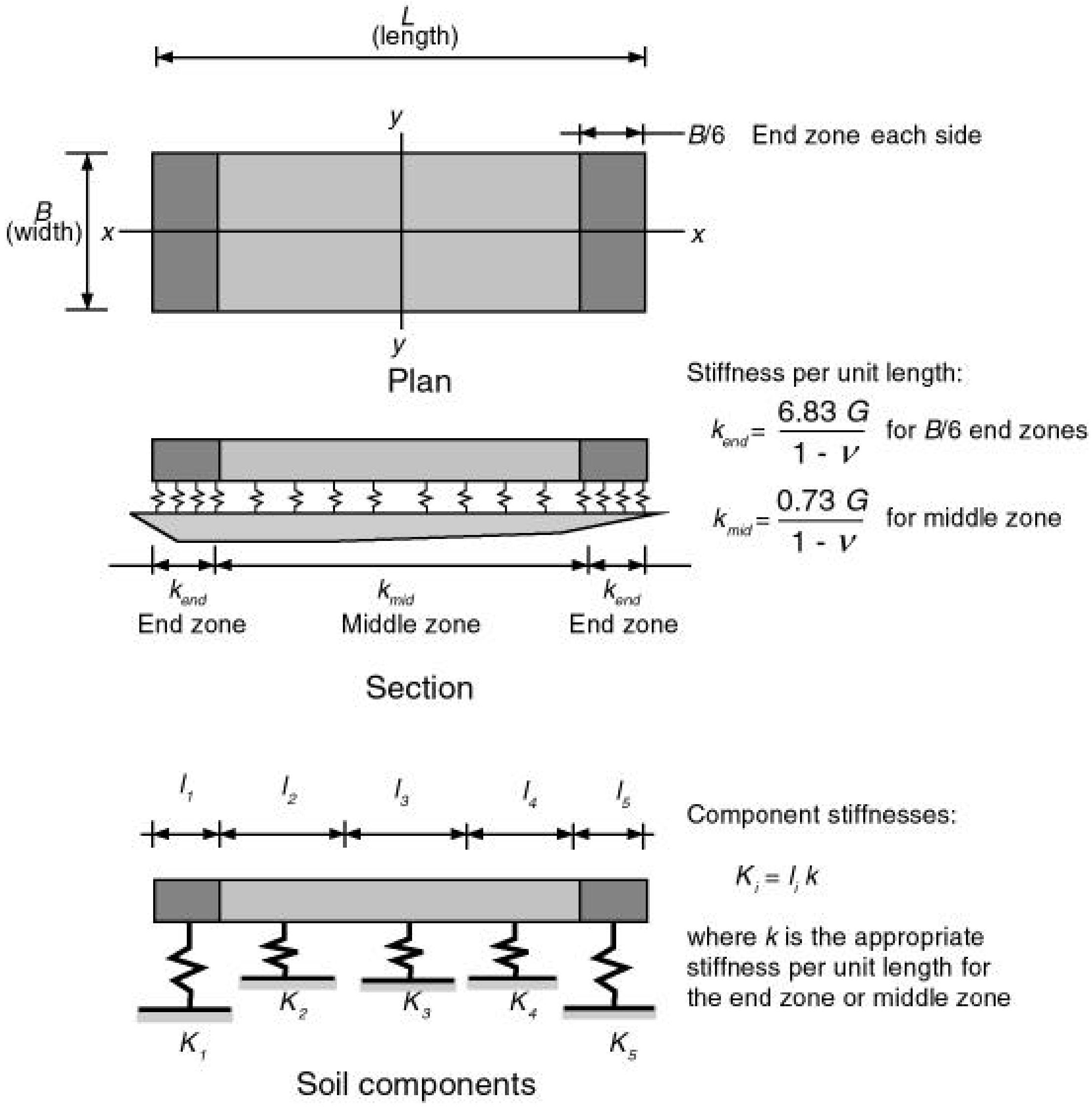


Figure 9-5 Vertical Stiffness Modeling for Shallow Bearing Footings

$$k_{sv} = \frac{1.3G}{B(1-\nu)} \qquad (9\text{-}9)$$

(b) Capacity parameters.

1. In the absence of moment loading, the vertical load capacity of a rectangular footing of width *B* and length *L* is

$$Q_c = q_c BL \qquad (9\text{-}10)$$

For rigid footings subject to moment and vertical load, contact stresses become concentrated at footing edges, particularly as uplift occurs. The ultimate moment capacity, M_c, is dependent upon the ratio of the vertical load stress, q, to the vertical stress capacity, q_c. Assuming that contact stresses are proportional to vertical displacement and remain elastic up to the vertical stress capacity, q_c, it can be shown that uplift will occur prior to plastic yielding of the soil when q/q_c is less than 0.5. If q/q_c is greater than 0.5, then the soil at the toe will yield prior to uplift. This is illustrated in Figure 9-6. In general, the moment capacity of a rectangular footing may be expressed as:

$$M_c = \frac{LP}{2}\left(1 - \frac{q}{q_c}\right) \qquad (9\text{-}11)$$

where:

P = vertical load

$$q = \frac{P}{BL}$$

B = footing width

L = footing length in direction of bending.

2. The lateral capacity of a footing should be assumed to be attained when the displacement, considering both base traction and passive pressure stiffnesses, reaches 2.0 percent of the thickness of the footing. Upper and lower bounds of twice and one-half of this value, respectively, also apply.

(3) Pile Foundations.

(a) General. Pile foundations, in the context of this paragraph, refer to those foundation systems that are composed of a pile cap and associated driven or cast-in-place piles, which together form a pile group. A single pile group may support a load-bearing column, or a linear sequence of pile groups may support a shear wall. Generally, individual piles in a group could be expected to be less than 2 feet (0.6m) in diameter. The stiffness characteristics of single large-diameter piles or drilled shafts are described in Paragraph 9-2c(4).

(b) Stiffness parameters. For the purpose of simplified analyses, the uncoupled spring model as shown in Figure 9-1b may be used where the footing in the figure represents the pile cap. In the case of the vertical and rocking springs, it can be assumed that the contribution of the pile cap is relatively small compared to the contribution of the piles. In general, mobilization of passive pressures by either the pile

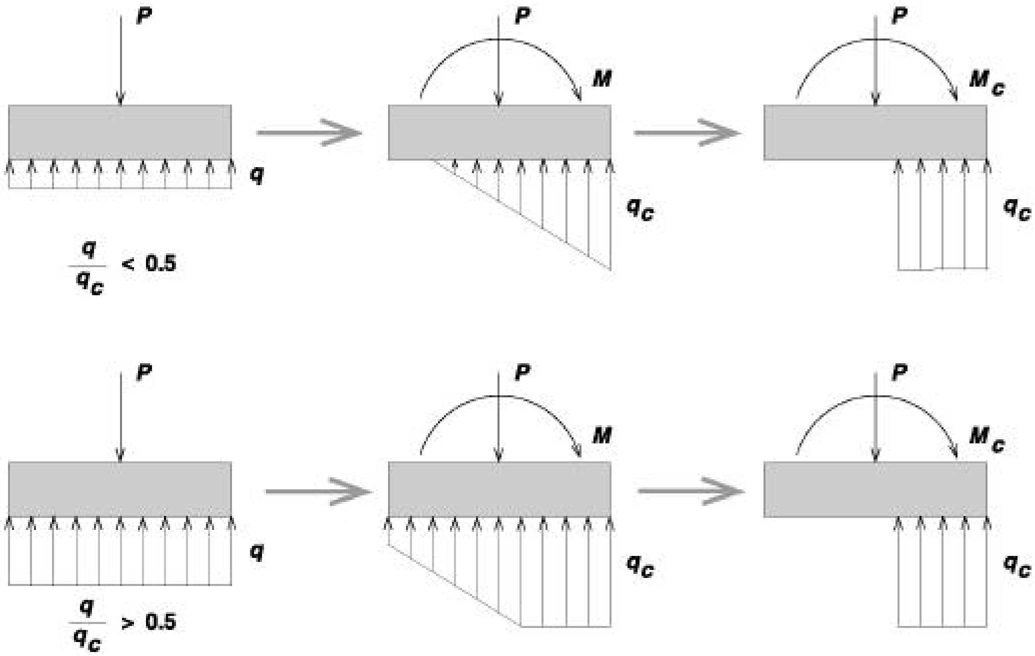


Figure 9-6 Idealized Concentration of Stress at Edge of Rigid Footings Subjected to Overturning Moment

caps or basement walls will control lateral spring stiffness; therefore, estimates of lateral spring stiffness can be computed using elastic solutions as described in Paragraph 9-2c(2)(a). In instances where piles may contribute significantly to lateral stiffness (i.e., very soft soils, battered piles) solutions using beam-column pile models are recommended. Axial pile group stiffness spring values, k_{sv}, may be assumed to be in an upper- and lower-bound range, respectively, given by:

$$k_{sv} = \sum_{n=1}^{N} \frac{0.5\,AE}{L} \text{ to } \sum_{n=1}^{N} \frac{2\,AE}{L} \qquad (9\text{-}12)$$

where:

A = cross-sectional area of a pile

E = modulus of elasticity of piles

L = length of piles

N = number of piles in group.

The rocking spring stiffness values about each horizontal pile cap axis may be computed by assuming each axial pile spring acts as a discrete Winkler spring. The rotational spring constant (moment per unit rotation) is then given by:

$$k_{sr} = \sum_{n=1}^{N} k_{vn} S_n^2 \qquad (9\text{-}13)$$

where:

k_{vn} = axial stiffness of the nth pile

S_n = distance between nth pile and axis of rotation.

Whereas the effects of group action and the influence of pile batter are not directly accounted for in the form of the above equations, it can be reasonably assumed that the latter effects are accounted for in the range of uncertainties expressed for axial pile stiffness.

(c) Capacity parameters. Vertical load capacity of piles (for both axial compression and axial tensile loading) should be determined and documented in the geotechnical investigation report. The investigation should be based on accepted foundation engineering practice using best estimate of soil properties. Consideration should be given to the capability of pile cap and splice connections to take tensile loads when evaluating axial tensile load capacity. The moment capacity of a pile group should be determined assuming a rigid pile cap, leading to an initial triangular distribution of axial pile loading from applied seismic moments. Full axial capacity of piles may, however, be mobilized when computing ultimate moment capacity, leading to a rectangular distribution of resisting moment in a manner analogous to that described for a footing in Figure 9-6. The lateral capacity of a pile group is largely dependent on that of the cap, as it is restrained by passive resistance of the adjacent soil material. The capacity may be assumed to be reached when the displacement reaches 1.0 percent of the depth of the cap in a manner similar to that for a shallow bearing foundation.

(4) Drilled shafts. In general, drilled shaft foundations or piers may be treated similarly to pile foundations. When the diameter of the shaft becomes large (>24 inches), the bending and the lateral stiffness and strength of the shaft itself may contribute to the overall capacity. This is obviously necessary for the case of individual shafts supporting isolated columns. In these instances, the interaction of the soil and shaft may be represented using Winkler-type models.

9-3. General Requirements.

a. Base. The base of the building is the level at which the earthquake motions are considered to be imparted to the structure. From the point of view of design, the base is the level at which the base shear is resisted. In a building without a basement, this is simply at grade, where footings develop lateral resistance. In a building with a basement, the base is at grade if grade-level framing or the upper portion of the basement wall is capable of developing the required lateral resistance, or at the basement level, if the lateral resistance cannot be developed at grade level. On sloping sites, the level at grade may be unrestrained at the downhill side, but restrained, like a basement, at the uphill side. The base of a building is determined by judgment, considering the mechanism for developing lateral resistance. The base should be taken at the highest level where the building can transmit lateral forces into the ground on all sides. Partial basements and sites with varying subsurface conditions are also potentially troublesome. The engineer should consider how the forces enter the substructure, and how they are transmitted into the ground. Simple three-dimensional free-body diagrams of whole substructures may be of great help in defining the design conditions.

b. Column Base. If a column is assumed to be fixed in the analysis of the superstructure, the foundation system must have the strength and stiffness required by this assumption.

c. Development of Forces into the Foundations. Foundations must be detailed to develop the horizontal and vertical components of seismic forces imparted by columns, shear walls, and braces. In instances where footing are subjected to lateral thrusts due to applied vertical loads, such horizontal thrust will be added to the lateral seismic force indicated above. An example of this case could be the outward thrusts on footings of a rigid gable bent due to applied vertical loads.

d. Interconnection of Foundation Elements. Foundation ties shall be provided as required by Chapter 7 of FEMA 302. The ties can be formed by an interconnecting grid network of reinforced concrete struts or structural steel shapes encased in concrete. As an alternative, a reinforced concrete floor slab, doweled to walls and footings to provide restraint in all horizontal directions, may be used in lieu of the grid network of ties. Slabs on grade will not be used as ties when a potential for liquefaction has been identified, or when significant differential settlement is expected between footings and slab. In such cases, slabs on grade will be cut loose from footings and made free-floating (note that the effective unsupported height of the wall is increased for this condition.) Strut ties placed below such slabs will be cushioned or separated from the slab such that slab settlement will not damage the slab or strut ties. Alternatively, it may be more economical to overexcavate the soil under the footings and recompact to control differential settlements under vertical loads, and to increase passive resistance of the sides of the footings under lateral loads so as to eliminate the need for footing ties. Slabs on ground when used as a foundation tie will have minimum reinforcing, according to ACI 318. As a minimum, a mat of #4 at 16 inches each way is recommended.

e. Overturning. The overturning moment at the base of the building is resisted by the soil through the foundation. The total load on the soil is not changed, but there is a change in the distribution of the soil pressure. For isolated spread footings, the design requirement is simply to provide for vertical components of the overturning moment in combination with the vertical forces due to dead and live loads. For wall footings, there may be enlarged footings under the boundary members, and these will have increased loads as indicated above for isolated footings, but there will also be loads on grade beams or other connecting elements.

f. Differential Settlement. Earthquake vibrations may cause consolidation or liquefaction of loose soils, and the resultant settlement of building foundations usually will not be uniform. For rigid structures supported on individual spread footings bearing on such material, excessive differential settlements can damage the superstructure. Stabilization of the soil prior to construction, or the use of piles, caissons, or deep piers bearing on a firm stratum, may be the solution to this problem.

9-4. Design of Elements.

a. General. The mechanism used for the transmission of horizontal forces may be friction between floor slab and ground; friction between bottom of footing and ground; and/or passive resistance of earth against vertical surfaces of pile caps, footings, grade beams, or basement walls. The overturning effects, which require a careful analysis of permissible overloads for the combined effect of vertical and lateral loads, must be considered in the foundation design. Although rocking of buildings about their foundations appears to have been beneficial in some instances, it is not permitted by this document because of the indeterminate nature of rocking as a means of energy dissipation. Net upward forces must be resisted by anchorage into the foundation. Stability against overturning must be provided for the short-time loading during an earthquake (or wind) without creating disparities in the foundation configuration that would result in significantly different foundation settlements due to gravity loads. These differential settlements could create more damage to the structure than the short-time deformations that might occur under the highly increased soil pressures due to earthquake effects.

b. Slabs on Ground. Slabs on ground are often thought of as nonstructural, but will in fact be nonstructural only if detailed to be unconstrained by adjacent elements. In seismic design, the slab on ground should be used as a connecting, tying, stiffening element by suitable details of joints and reinforcing in the slab and at the edges of the slab.

c. Grade Beams. Grade beams may be used to stiffen spread footings where columns are intended to have fixed bases; grade beams may also develop lateral resistance in passive pressure on their sides, especially if stiffened by an integral slab on ground. Passive-resistance values vary greatly with type of soil and depth. Adequacy of passive resistance should be determined by the geotechnical engineer. Passive resistance or lateral bearing values are permitted only where concrete is deposited directly against natural ground, or the backfill is well compacted. Passive resistance should not be used where the lateral bearing surface is close to an excavation, unless such excavation is carefully backfilled with well-compacted material. The shear capacity of the soil between such bearing surface and open or poorly compacted excavation or a similar depression may be inadequate to provide the needed resistance.

d. Basement Walls. Basement walls can develop passive pressure for normal forces. The comments on passive pressure for grade beams apply.

e. Spread Footings. Spread footings resist vertical loads through bearing pressure on the bottom, and resist horizontal loads through friction on the bottom and passive pressure on the sides.

f. Wall Footings. Wall footings resist lateral loads through friction on the bottom.

g. Piles. Piles driven into soft surficial soils must transfer the base shear into stiffer soils at lower levels. This involves bending of the piles. Criteria for design should be obtained from the geotechnical engineer. Where subsurface conditions vary over the site, the effective lengths of piles in bending may vary. The resulting variation in relative rigidity causes some piles to carry more lateral load than others, and must be considered in the foundation design. Passive pressure on the vertical surfaces of the pile cap may be a more effective method of shear transfer. For pile caps in weak soils, the use of select compacted backfill will enhance the lateral load capacity.

h. Batter Piles. The use of batter piles should be avoided. Their greater lateral stiffness relative to the vertical piles attracts most of the lateral forces to themselves, resulting in an unbalanced lateral-load-resisting system. Because the inclination of the batter piles is usually small, very large vertical components of force are developed between the vertical and adjacent batter piles. The pile cap must be detailed to accommodate these forces, and the caps may need to be stiffened by horizontal grade beams to prevent rotation under these forces.

i. Foundation Ties. Ties will be designed to carry an axial tension and compression horizontal force equal to 10 percent of the larger column load. The minimum tie will be 12 inches by 12 inches (305mm x 305mm) , with four #5 (15M) longitudinal bars and #3 (10M) ties at 12 inches (305mm) on center.

j. *Retaining Walls.* Building walls retaining soil should be evaluated for the effects of seismic earth pressures. The seismic earth pressure acting on a building wall retaining nonsaturated, level soil above the groundwater table may be approximated as:

$$Dp = 0.4k_h\gamma_t H_{rw} \qquad (9\text{-}15)$$

where:

Dp = additional earth pressure due to seismic shaking, which is assumed to be a uniform pressure

k_h = horizontal seismic coefficient in the soil, which may be assumed equal to 0.5 S_{DS}/2.5

g = the total unit weight of soil

H_{rw} = the height of the retaining wall.

The seismic earth pressure given above should be added to the static earth pressure to obtain the total earth pressure on the wall. The expression in Equation 9-13 is a conservative approximation of the Mononabe-Okabe formulation. Seismic earth pressures much higher than summarized above may develop on walls that are required to develop passive pressures to resist lateral forces. In such cases, static passive earth pressure formulations, neglecting inertia forces in the soil, may be used to estimate the magnitude of total (static plus seismic) earth pressures on the wall. A triangular pressure distribution may be assumed.

k. *Mixed Systems.* When subsurface conditions vary significantly across a site, it is sometimes effective to use mixed systems, e.g., combinations of drilled piers and spread footings. Geotechnical consultation is especially important for mixed systems in order to control differential settlements. The difference in lateral stiffnesses between the spread footings and drilled piers must be considered in the foundation earthquake design. Nominal values of the soil springs, determined in accordance with the requirements of this chapter, may be used in the analysis.

9-5. Acceptance Criteria.

a. *Performance Objective 1A.* The response modification factors, *R*, for Performance Objective 1A, shall be in accordance with the structural system identified in Table 7-1. The design of the foundation shall be in accordance with Chapter 7 of NEHRP as modified by this chapter.

b. *Enhanced Performance Objectives.*

(1) Linear elastic analyses with *m* factors. Structural foundation components should be considered to be force-controlled, and their lower-bound capacity, Q_{CL}, will be the nominal capacity, in accordance with FEMA 302, multiplied by the appropriate capacity reduction factor, *N*. If soil springs are used in the analyses, the nominal stiffness coefficients prescribed in this chapter are to be multiplied by 0.5 for Life Safety, 1.0 for Safe Egress, and 2.0 for Immediate Occupancy performance levels.